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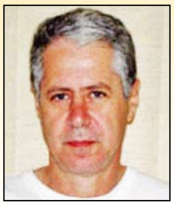
Reliability Evaluation of Reinforced Concrete Pipes in Crack Opening Limit State

Avaliação da Confiabilidade de Tubos de Concreto Armado no Estado Limite de Fissuração



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Abstract

Structural reliability theory is used in this paper to verify the capability of Brazilian code on design of concrete structures (NBR 6118:2003) concerning the evaluation of crack width in reinforced concrete pipes. Two limit state equations are defined in terms of crack opening. The First Order Second Moment and Monte Carlo simulation methods are used in the reliability analysis. In an initial reliability analysis, problem parameters that have the largest contributions in failure probabilities are identified. A parametric analysis is performed in these variables, in order to study their influence in failure probabilities. The study shows that the formulation of NBR 6118:2003 leads to non-uniform reliability, for a constant safety factor. This means that the unitary safety coefficient specified by the code for the cracking limit state does not reflect the uncertainty in the tubes resistance parameters.

Keywords: Pipe, Reinforced Concrete, Reliability, Crack.

Resumo

Com a teoria da confiabilidade avaliam-se as duas formulações apresentadas pela norma de projeto de estruturas de concreto NBR 6118:2003 para a estimativa da abertura de fissuras em tubos de concreto armado. Os métodos de confiabilidade *FOSM* (método de primeira ordem e segundo momento) e o método de simulação de Monte Carlo com amostragem por importância são utilizados. Uma primeira análise de confiabilidade revela as variáveis de projeto com maior contribuição nas probabilidades de falha. Uma análise paramétrica é realizada nestas variáveis, de maneira a identificar a influência destas na confiabilidade dos tubos. O estudo mostra que as formulações da NBR 6118:2003 levam a valores não uniformes para o índice de confiabilidade, para um mesmo fator de segurança. Isto significa que o coeficiente de segurança unitário especificado em norma para o estado limite de fissuração não reflete a incerteza nos parâmetros de resistência do tubo.

Palavras-chave: Tubo, Concreto Armado, Confiabilidade, Fissura.

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1. Introduction

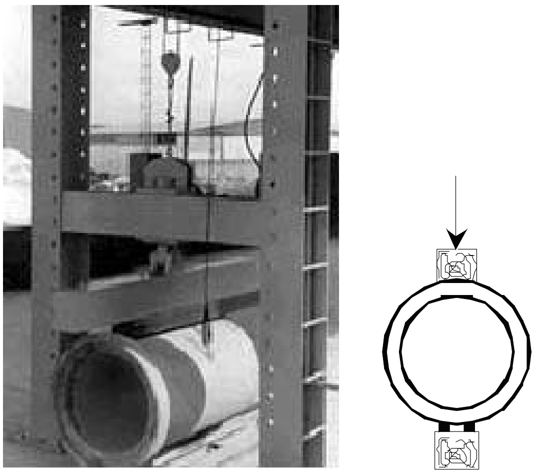
The structural design of buried pipes must meet ultimate and service limit states, verified from the internal forces. The difficulty in the evaluation of these internal forces is due to the fact that they depend on ground pressure over pipe walls, and this pressure depends on the form of installation (by ditches, landfill or driven) and on the settlement of the pipe (form of base and compaction of lateral earth fill).
In the design of buried pipes, the Marston-Spangler's procedure is usually employed (ZAIDLER [1]). This procedure involves determining the resultant of vertical operating loads in the pipe, using an equivalence factor that correlates the behavior of the pipe in the field and in standard test situations.
Amongst existing standard tests, the diametral compression test is one of the most widely used, due to its simplicity. This test is shown schematically in Figure 1.
An equivalence factor α_{eq} is used to determine the diametral compression force *F* in the standard test, to produce the same bending moments in the tube as the resultant of vertical loads acting on the pipe in situ:

$$F = \gamma(q + q_m)/\alpha_{eq} \tag{1}$$

where:
q is the resultant of vertical loads of soil;
q_m is the resultant of variable vertical loads;
α_{eq} is the equivalence factor;
γ is the safety factor.

According to NBR 8890:2003 [2], the safety factors applied to the vertical load expression are given by $\gamma = 1{,}0$ for the cracking limit state and $\gamma = 1{,}5$ for the ultimate limit state.

Figure 1 – Diametral compression test



The ultimate (collapse) load is the maximum force achieved in the diametral compression test, which leads to the ultimate limit state for the pipe. The service load is one for which a crack with opening of 0,25 mm will appear in the pipe, with a length of 300 mm or more, which corresponds to the service (cracking) limit state.
According to the force resisted in the diametral compression test, NBR 8890:2003 [2] divides the pipes in classes, according to the cracking and ultimate load of the pipe. The code sets the requirements and tests for the acceptance of simple and reinforced circular pipes destined for pluvial waters and sewage drainage.
NBR 6118:2003 [3] presents two formulas for calculating the characteristic value of crack opening. The aim of this work is to evaluate the reliability of these semi-empirical formulations for circular reinforced concrete pipes. Specifically, pipes used in pluvial water drainage and reinforced with welded steel meshes are addressed in the study. First, the main mechanical and geometrical parameters that influence reliability of the pipes are evaluated. Afterwards, a parametric analysis is realized in these variables.
This study is motivated by the absence of reliability analyses in the literature dealing with the cracking limit state of circular reinforced concrete pipes.

2. Crack opening limit state

The verification of crack opening can be made following procedures indicated in NBR 6118:2003 [3]. This code provides the following expressions to determine the magnitude of crack opening:
The parameters in eq. (2) and (3) have the following notation:

$$w_a = \frac{\phi}{12{,}5 \cdot \eta} \frac{\sigma_s}{E_s} \frac{3 \cdot \sigma_s}{f_{ctm}} \tag{2}$$

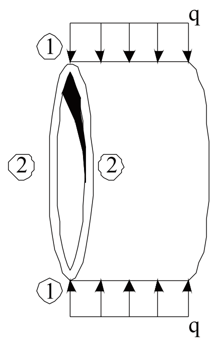
$$w_s = \frac{\phi}{12{,}5 \cdot \eta} \frac{\sigma_s}{E_s} \left(\frac{4}{\rho_r} + 45\right) \tag{3}$$

φ is the wire diameter of the reinforcing steel mesh;
η is the adhesion coefficient of the reinforcing wire mesh in tension;
E_s is the steels elasticity modulus;
f_{ctm} is the average concrete resistance in tension;
w_a e w_s are the asystematic and systematic cracking, respectively;
σ_s is the tensile stress in the reinforcement mesh in tension, which can be calculated from:

$$\sigma_s = \frac{M_d}{0{,}9 \cdot d \cdot A_s} \tag{4}$$

where:
d is the section height;
M_d is the distributed moment corresponding to the cracking load, per linear meter;

Figure 2 – Idealized structural behavior for band of unit width; sections 1 and 2 correspond to crown/base and flank sections, respectively



A_s is the area of the reinforcing wire mesh in tension, per linear meter;
ρ_r is the geometric rate of the reinforcement wire with respect to area A_{cr}.

$$\rho_r = \frac{A_{si}}{A_{cr}} \quad (5)$$

where:
A_{cr} is the area of concrete that involves the wire of the steel mesh (NBR 6118:2003 [3]);
A_{si} is the area of the wire of the reinforcement mesh submitted to tension.

In Brazil, the reinforcement steel meshes have only been produced with corrugated wire. In the absence of precise indications for the value of η, it is recommended to use $\eta = 2{,}25$ for evaluation of crack opening, which corresponds to the case of high adhesion.

The smallest value between w_a and w_s is used in the evaluation of crack opening. This value must be limited to 0.25 mm, which corresponds to the cracking load in the diametral compression test. It is noted that there is reasonable uncertainty in the definition of this parameter, to which the NBR 6118:2003 design code [3] refers to as an "order of magnitude" value.

2.1 Internal forces

The pipe is designed to withstand the diametral compression test. In this situation, the pipe is subject to an uniformly distributed load along its axis. Considering a state of plane strain, the pipe can be defined through a band of unit width, as shown in Figure 2.
The pipe is analyzed at two reference sections: the crown/base and flank sections, where internal forces and displacements are maximum. Using the elastic theory applied to thin rings, the internal forces sketched in Figure 3 are obtained.

3. Structural reliability

The fundamental problem of structural reliability can be formulated from the relationship between load action effect S and resistance R. The failure event occurs when $R - S < 0$ or $R/S < 1$. FREUDENTHAL *et al.* [4] and ANG & TANG [5] define the failure probability as the integral over the domain of the product of functions $F_R(s)$ and $f_S(s)$, as shown in Equation 6 and Figure 4. The failure probability defined in Equation 6 assumes independence between the S and R variables.

$$p_F = \int_0^{+\infty} F_R(s)\ f_S(s)\ ds \quad (6)$$

The black area in Figure 4 represents the failure probability p_F, which is proportional to the region of interference between the resistance and load effect curves. The bigger this interference area is, the greater the failure probability.
Generalizing for a problem involving n random variables, the failure

Figure 3 – Diagram of internal forces produced by opposed diametrical actions

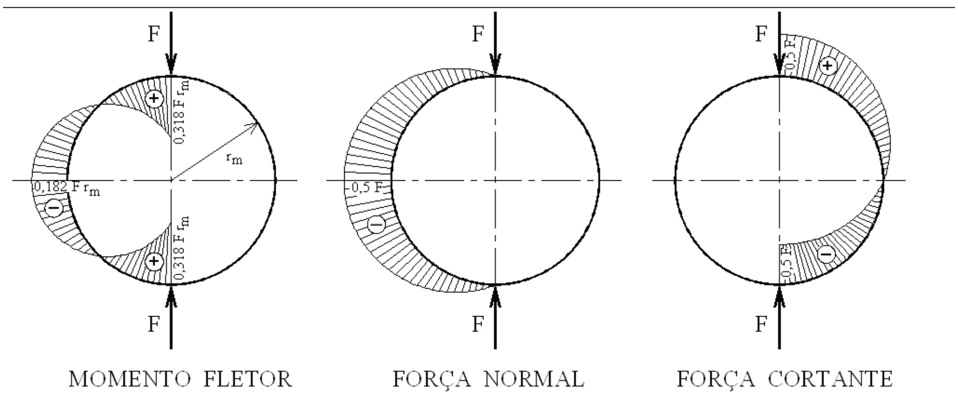
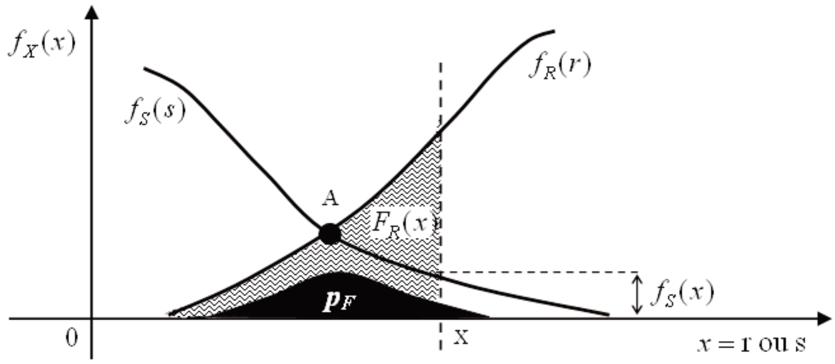


Figure 4 – Density functions $f_R(r)$ and $f_S(s)$ – Source: SILVA (2006)



probability is evaluated by Equation 7, where $f_X(x)$ is the joint probability density function and D_f is the failure domain, defined by limit state equation g(**X**), written in function of design variables **X**.

$$p_F = \int_{[g(X)<0]} f_{X_1,X_2,\ldots,X_n}(x_1,x_2,\ldots,x_n)\ dx_1dx_2\ldots dx_n \quad (7)$$

Methods used to evaluate equation 7 are differentiated by the approximations made in evaluating $f_X(x)$ and D_f. The joint density function $f_X(x)$ is determined on the basis of existing information, basically the marginal distributions functions (for instance, $f_S(s)$ e $f_R(r)$) and the correlation coefficient between pairs of variables (ρ_{RS}).

The First Order Second Moment met od (FOSM) has the following main characteristics:

a) the limit state equation is approximated by a linear function;

b) construction of the joint function $f_X(x)$ is based on second order moments (mean and covariance); that is equivalent to assume variables with normal distribution, possibly correlated.

The First Order Reliability Method (FORM) distinguishes itself from FOSM because all the available statistical information on the random variables is used. The limit state equation is still approximated by a linear function.

The failure probability can also be evaluated by means of Monte Carlo simulation. The Monte Carlo method is largely used because of its simplicity. It consists in a repetition of deterministic solutions, based on samples generated in accordance to the probability distributions of the problems random variables. It is common to use the Monte Carlo method to verify other approximate solutions, like FOSM and FORM.

In the Monte Carlo method, it is also possible to use techniques to reduce the number of required samples, especially when failure probabilities are small. One of these techniques is called importance sampling, which translates the sampled points towards the failure domain, avoiding excessive simulation away from the failure domain.

From the failure probabilities defined in equations 6 and 7, one can define the reliability index *β*, given by: $\beta = -\Phi^{-1}(p_F)$. In this expression, $\Phi^{-1}(.)$ is the inverse of the Standard Gaussian cumulative distribution function.

The reliability index *β* can be compared to the so-called central safety factor (*FS*), following equations 8 and 9 (ref. [6]). These equations apply to a situation involving two correlated Gaussians basic variables: resistance and load effect. In equations 8 and 9, *FS* is the relationship between the means of resistance and load effect. The reliability index *β* is the ratio between the mean and the standard deviation of the safety margin.

$$FS = \frac{m_R}{m_R - \beta\ \sqrt{\sigma_R^2 + \sigma_S^2 - 2\ \rho_{RS}\ \sigma_R\ \sigma_S}} \quad (8)$$

$$\beta = \frac{1 - 1/FS}{\sqrt{v_R^2 + (1/FS)^2\ v_S^2 - \dfrac{2\ \rho_{RS}\ \sigma_R\ \sigma_S}{m_R^2}}} \quad (9)$$

In these equations:

m_R is the mean resistance;

σ_R and σ_S are the resistance and load effect standard deviations, respectively;

v_R and v_S are the *R* and *S* coefficients of variation, respectively;

ρ_{RS} is the correlation coefficient between *R* and *S*.

It can be noticed that the relationship between *β* and FS is strongly non-linear, and depends on the statistical moments of *R* and *S*.

4. Methodology of analysis

An initial reliability analysis was performed to reveal the most important parameters in the reliability of the concrete pipes. After this study, parametric analyses were performed, varying the parameters of the most influential variables. These analyses consist in evaluation of the central safety factors *FS* associated reliability indexes *β*, considering two explicit limit state functions. The computational program used in the analyses was developed by BECK[7].

4.1 Limit state equations

According to the formulation presented in NBR 6118:2003 [3] to estimate crack opening in reinforced concrete structures, two limit state functions are obtained. From these equations, it is possible to evaluate the probability of a given concrete pipe not reaching the minimum cracking load, once specified the pipes class.
From the expressions of NBR 6118:2003 [3] that indicate the force necessary to cause mean crack opening in the diametral compression test, the following limit state equations (EEL) are obtained in terms of the force *F* applied in the test:

$$\text{EEL1:}\quad g(X) = F_1 - F \tag{10}$$

$$\text{EEL2:}\quad g(X) = F_2 - F \tag{11}$$

where:

$$F_1 = \frac{0{,}9.d.A_s}{C.R_m}.\sqrt{\frac{12{,}5.w\eta.E_s.f_{ctm}}{3.\phi}} \tag{12}$$

$$F_2 = \frac{0{,}9.12{,}5.w\eta.E_s.d.A_s}{C.\phi.R_m.\left(\frac{4}{\rho_{ri}}+45\right)} \tag{13}$$

and:
R_m is the mean radius of the pipes circular cross-section;
C is the constant that varies according to the bending moments diagram in the diametral compression test. It is equal to 0.318 if the rounding of bending moments in the crown are not considered (EL DEBS [8]);
F is the minimum force for a crack opening of 0.25 mm, as suggested in NBR 8890:2003 [2], for a specified pipe class.
The *F1* and *F2* terms represent two resistance conditions for the cracking limit state of reinforced concrete pipes.
In order to perform the reliability analysis in the cracking limit state, the following assumptions were considered:

a) Until the crack opening of 0.25 mm, bending moments grow linearly with the load. This assumes no significant load redistributions due to nonlinear behavior. This hypothesis is reasonable when working with service loads.
b) For a preliminary evaluation, all random variables are assumed to follow a normal distribution.
c) The basic variables are considered non-correlated, because no correlation information is available.
d) Expressions 12 and 13 were determined for the crown section, where bending moments are more critical in relation to the appearance of cracks, as shown in Figure 3.

4.2 Data used in reliability analysis

In the design of reinforced concrete pipes of nominal diameters less than 800 mm, usually circular reinforcement is used. In this study, a pipe with nominal diameters of 800 mm is considered.
Based on information from a Brazilian manufacturer, pipes of 800 mm have the following characteristics: the walls thicknesses (*h*) have standard value of 72 mm, the sections useful height (*d*) has a mean of 45 mm and the concrete used in pipe production has characteristic compression strength (f_{ck}) of 35 MPa.
Concrete variability is defined in accordance with NBR 12655:1996 [9]. Following this code, concrete resistance depends on preparation conditions, and the following values are obtained for the standard deviation of concrete strength: 4 MPa; 5.5 MPa and 7 MPa.
According to NBR 8890:2003 [2], the axial force to be resisted by the pipe in the diametral compression test, or minimum cracking load for pluvial drainage use, is 32 kN/m.
The values of crack opening, the conformation coefficient for the tensile reinforcement surface and the diameter of the reinforcement mesh wire are: $w = 0{,}25$ *mm;* $\eta = 2{,}25$ e $\varphi = 7{,}1$ *mm*, respectively.
Since not all random variable statistics can be observed from experimental data, it is necessary to adopt literature values and the authors experience to infer such statistics.

5. Results and discussion

5.1 Preliminary reliability analysis

In a preliminary reliability analysis, the following random variables are considered: useful height (*d*), concrete compression strength (f_c), pipe wall thickness (*h*), diametral compression force in the standard test (*F)* and steels elasticity modulus (E_s). Table 1 shows the parameters used for each of the problems random variable.
The FOSM method is used in this study to obtain reliability indexes. Subproducts of this analysis are sensitivity coefficients, which indicate which variables give the most contribution to the evaluated failure probability.
Table 2 shows, for each limit state equation, the design points, the sensitivity coefficients, the reliability indexes and the failure probabilities evaluated. Since the purpose of this investigation is to evaluate safety and reliability, both limit state functions are considered, and not only that leading to the largest crack opening.
Figure 5 shows the influence of random variables in each limit state

Table 1 – Random variable data

V.A.	Mean	Standard deviation	C.O.V.(%)
X1 = d	45mm	4.5mm	10
X2 = f_c	41.6MPa	4MPa	9.62
X3 = h	72mm	7.2mm	10
X4 = F	32kN	0.32kN	1
X5 = E_s	2.1e5MPa	50.4e2MPa	2.4

Table 2 – Design point, sensitivity and reliability for EEL1 and EEL2

V.A.	EEL1 Design Point	EEL1 Sensitivity	EEL2 Design Point	EEL2 Sensitivity
X1 = d	2.772	92.38%	4.179	54.53%
X2 = f_c	3.749	6.52%	4.158	0%
X3 = h	7.340	0.24%	7.656	43.18%
X4 = F	32.07	0.35%	32.017	0.34%
X5 = E_s	20855.9	0.51%	20932.2	1.95%
β_{FOSM}	3.995		0.964	
$p_{F,FOSM}$	3.227e-5		1,674e-1	

equation (EEL1 and EEL2) for the data shown in Table 1. Note that there is no influence of variable f_c in EEL2, because this variable does not belong to this equation (equation 13).

In EEL1 the most important variables are useful height (*d*) and concrete compression resistance (f_c). In EEL2 the most important variables are useful height (*d*) and pipe wall thickness (*h*).

5.2 Parametric analysis

The sensitivity coefficients obtained in the preliminary reliability analysis revealed that the largest contribution to pipe failure probability is due to useful height and concrete strength random variables. Parametric analyses are carried out in these variables, by varying the standard deviation of these variables. The First Order Second Moment method (FOSM) and Monte Carlo simulation with importance sampling are used in this analysis.

According to the variances of useful height (*d*) and concrete strength (f_c), the parametric analysis is composed of 6 cases, as shown in Table 3. For all cases, the pipe wall thickness (*h*) has a fixed standard deviation of 7.2 mm.

Tables 4 and 5 summarize the reliability analysis results for each limit state equation, EEL1 and EEL2, respectively. The evaluated reliability parameters are: reliability index and failure probability evaluated by FOSM (β_{FOSM}, $p_{F,FOSM}$) and by Monte Carlo simulation with importance sampling (β_{MCAI}, $p_{F,MCAI}$); the central safety factor and reliability index evaluated from equations 8 and 9. The Monte Carlo simulation uses 1000 sampling points.

The two formulas provided in NBR 6118/2003 for evaluation of crack opening represent a series system, where failure is characterized by the smallest load obtained in equations 10 and 11.

Tables 4 and 5 show that no difference is noted for the central safety factor related to EEL1 and EEL2, when compared to the

Figura 5 – Sensitivity coefficients of random variables for limit state equations EEL1 and EEL2

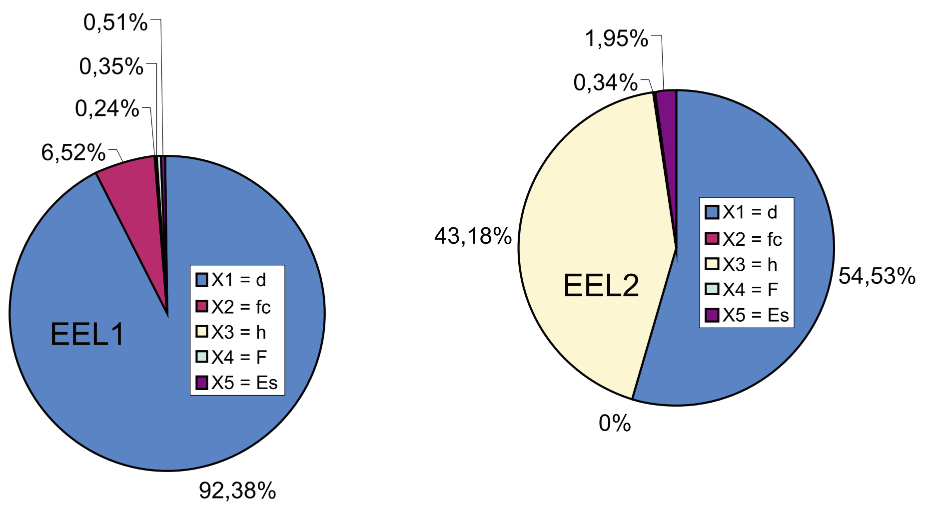


Table 3 – Random variable data for parametric analysis

Case	Standard deviation d (mm)	Standard deviation f_c (MPa)	C.O.V. (%) d	C.O.V. (%) f_c
Case 1	4.5	4	10	9.62
Case 2	4.5	5.5	10	12.48
Case 3	4.5	7	10	15.05
Case 4	9	4	20	9.62
Case 5	9	5.5	20	12.48
Case 6	9	7	20	15.05

reliability index variation as measured by FOSM, by Monte Carlo simulation or by equation 9.

Comparing the reliability indexes presented in Tables 4 and 5, it is observed that β_{FOSM} and β_{MCAI} are greater than $\beta_{equation9}$. This, however, was not observed in Table 4 for the cases 1, 2 and 3 of the Monte Carlo simulation.

According to results presented in Tables 4 and 5, an increase in the coefficients of variation of *d* and *fc*, causes decrease in β_{FOSM}, β_{MCAI} and $\beta_{equation9}$, as could intuitively be expected. It is also observed in Table 4 that the variation in reliability index is more sensitive to the uncertainty in *d*, as observed in cases 4, 5 and 6.

It is valid to emphasize that the reliability indexes obtained in this study reflect the variability of the random variables in each case, which does not occur with the central safety factor, which remained constant for EEL1 and EEL2. Generally, specifying only the central safety factor to the designer will lead to variations in reliability indexes. The associated reliability index should be supplied, because this allows the uncertainty to be properly taken into account. This conclusion can also be found in other structural engineering applications, as in AOKI [10] and SILVA [11].

The design points in physical space are shown in Figure 6 for EEL1 and EEL2. In this figure, μ_X represents the mean point, X1* and X2* are the design points associated to EEL1 and EEL2, respectively. The lines that pass through X1* and X2* are the failure surfaces. Figure 6 shows that limit state equation EEL2 better represents the crack limit state in this analysis. Of course, this conclusion is limited to the pipe configuration investigated in this study.

Results presented in Tables 4 and 5 can be compared with recommendations by the Eurocode 1 [12] and CEB [13]. Eurocode 1 recommends reliability index values of: 1.5 for service limit state and 3.8 for ultimate limit state. The CEB [13] also presents some suggestions in accordance with required safety classes, as shown in Table 6. The values presented in this table can be used as reference in structural design, that is, the structure is designed or verified so that its reliability matches the suggested values. In a comparison tio the suggested values, the following conclusions can be stated:

a) in all analysis, EEL1 satisfies the Eurocode 1 [11] suggestion for service limit state; the same is not true for EEL2;

b) analysis cases 1, 2 and 3 (Table 4) satisfy safety levels 1 and 2 as recommended by the CEB [13], which is not true for EEL2 and cases 4, 5 and 6.

6. Conclusions

This paper presents a contribution to the study of reinforced concrete pipes in the crack opening limit state, a topic not extensively covered in the literature.

In the formulation of NBR 6118:2003 [3] for evaluation of characteristic values of crack opening in reinforced concrete pipes, the most important variables are useful height, concrete compressive strength and pipe wall thickness.

This study showed that for the same central safety factor, the formulations of NBR 6118:2003 lead to distinct reliability indexes, depending on the uncertainty in the important random variables. Hence, design based on a central safety factor does not guarantee uniform reliability. The same is true for the design of concrete pipes in the crack opening limit state. The safety coefficient established in design code for this limit state ($\gamma = 1{,}0$) does not reflect the uncertainty in resistance parameters of the tube.

7. Acknowledgements

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Table 4 – Results of parametric analysis for EEL1

Case	β_{FOSM}	$p_{F,FOSM}$	β_{MCAI}	$p_{F,MCAI}$	FS	$\beta_{equation9}$
Case 1	4.018	2.931e-5	3.633	1.399e-4		3.863
Case 2	3.884	5.127e-5	3.581	1.711e-4		3.662
Case 3	3.613	1.515e-4	3.360	3.892e-4		3.440
Case 4	2.054	1.999e-2	2.042	2.057e-2	1.705	2.031
Case 5	2.041	2.061e-2	2.043	2.050e-2		2.000
Case 6	2.024	2.148e-2	2.009	2.225e-2		1.961

Table 5 – Results of parametric analysis for EEL2

Case	β_{FOSM}	$p_{F,FOSM}$	β_{MCAI}	$p_{F,MCAI}$	FS	$\beta_{equation9}$
Case 1						
Case 2	0.975	1.646e-1	0.973	1.652e-1		0.891
Case 3					1.182	
Case 4						
Case 5	0.595	2.757e-1	0.636	2.624e-1		0.565
Case 6						

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Table 6 – Reliability index according to safety classes, Source: CEB (1991)

Safety Level	1	2	3
Service Limit State	2.5	3.0	3.5
Utimate Limit State	4.2	4.7	5.2